Steel Interchange is an open forum for *Modern Steel Construction* readers to exchange useful and practical professional ideas and information on all phases of steel building and bridge construction. Opinions and suggestions are welcome on any subject covered in this magazine.

The opinions expressed in *Steel Interchange* do not necessarily represent an official position of the American Institute of Steel Construction, Inc. and have not been reviewed. It is recognized that the design of structures is within the scope and expertise of a competent licensed structural engineer, architect or other licensed professional for the application of principles to a particular structure.

If you have a question or problem that your fellow readers might help you to solve, please forward it to us. At the same time, feel free to respond to any of the questions that you have read here. Contact *Steel Interchange* via AISC's Steel Solutions Center:

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DESIGNING FOR LARGE COPE DEPTHS

Beams coped at both flanges are constrained to $c \leq 2d$ and $d_c \leq 0.2d$ in the 2nd Edition *LRFD Manual*, Vol. II, where c is the length of cope, d is the depth of the beam, and d_c is the depth of cope. How can beams with deeper copes ($d_c > 0.2d$) be designed?

Question sent to AISC Steel Solutions Center

Local web buckling could become a problem for too deep of a cope. The 3rd Edition *LRFD Manual* has a conservative procedure that can be used. This procedure, which is based on the same plate buckling model used for the non-deep copes, is found on page 9-9 of the *Manual*.

Sergio Zoruba, Ph.D.
AISC Steel Solutions Center
Chicago

LEED CERTIFICATION

We have a customer that is interested in pursuing LEED certification on their building. They also like the idea of using recycled materials and would like to have some type of certification to the recycled content of steel. Do you have any information I can pass on to this customer?

Question sent to AISC Steel Solutions Center

Check out www.aisc.org/sustainability. There is a good article there, which outlines how structural steel can be used towards a LEED (Leadership in Energy and Environmental Design) rating. There are also sample letters on the page that certify the post-consumer and post-industrial recycled material content for our member mills, and provide contact information at those mills, should further documentation be required. Certification letters from non-member mills will be supplied soon.

Chris Hewitt
American Institute of Steel Construction
Chicago

CRITICAL BUCKLING STRESS

Please refer to 3rd Edition *LRFD Manual*, Example 9.1 on pages 9-17 thru 9-20. How can the critical buckling stress ϕF_{bc} be greater than $0.9F_y$? In the example (F_y = 50 ksi), ϕF_{bc} = 77.3 ksi which is much greater than $0.9 \times 50 = 45$ ksi? Being accustomed to using ϕ for flexure of 0.9 (when in combination with F_y), why is this different? On page 9-7, under "Local Buckling," the wording is: *connection elements are thick enough that local buckling will not limit the design strength for flexure.* Does this mean that a "cap" of $0.9F_y$ does not apply?

Question sent to AISC Steel Solutions Center

There are two checks that are made: flexural yielding (a function of F_y) and local buckling (a function of F_{bc}). Since the design strength is calculated as $\phi F \times S/e$, where F takes on the value of either F_y or F_{bc}, the design strength is limited by the smaller of ϕF_y and ϕF_{bc}. So, in the example at hand, even though plate buckling theory suggests that local buckling won't occur until a stress of 77.3 ksi, this could never happen since flexural yielding would have already occurred.

Keith Mueller, Ph.D.
AISC Steel Solutions Center
Chicago

EXPANSION ANCHORS AND WASHERS

I often detail oversized (OVS) holes at expansion anchors as requested by the erector to facilitate field drilling. I also sometimes use OVS holes at lightly loaded connections that may be difficult to align in the field. The RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts* calls for hardened washers at OVS holes, plus, the connection has to be designed as slip-critical (which requires no paint at the connection area). Questions:

1. **Should I specify hardened washers on the plan and detail sheets, or is this something that the erector knows to do?**
2. **Do I need to detail no paint areas around the connection outer plies?**

Question from the steel-detail@yahoogroups.com list-server

Oversized holes in steel-to-steel connections are different than the larger hole sizes used in base plates and similar items that attach steel to concrete. The requirements you summarized are specific to steel-to-steel connections and do not apply to base plates.

But—base plate holes are larger and have washer requirements of their own. The washer requirements depend upon the type of load transfer. If a column will be in compression only, the washers are normally controlled by erection considerations. If there is a moment at the base, uplift and/or shear, the washer requirements may be very critical to the performance of the structure and the engineer should specify what is required.

Charles Carter, P.E., S.E.
American Institute of Steel Construction
Chicago

ASTM A572 GRADE 50 AND ASTM A992

Are there any differences between steel grades ASTM A572 Gr. 50 and ASTM A992?

Question sent to AISC Steel Solutions Center

There are differences, although the two materials are similar. ASTM A992 should be specified for all W-shapes used today. It is similar to ASTM A572 Gr. 50, but has better controls on chemistry and mechanical properties. It includes minimum values for yield and tensile strengths, a maximum ratio for yield strength to tensile strength, and a maximum carbon equivalent value. It is also less expensive than ASTM A572 Gr. 50 (and ASTM A36) for wide flange shapes. Note, however, that ASTM A992 is not currently used in the production of any shapes besides W-shapes

Bill Liddy
AISC Steel Solutions Center
Chicago

BENDING LIMITS

What are the maximum and minimum curved radii of HSS and W-shapes?

Question sent to AISC Steel Solutions Center

Limits on radii of curved shapes are essentially a function of the capabilities of the bender. Such deformations generally cannot approach deformations that would damage the material as can be the case for bent plates. As a result, AISC does publish guidelines for bending plates in the 3rd Edition *LRFD Manual*.

Cold bending guidelines for shapes are also found in the 3rd Edition *LRFD Manual* on page 2-39. They are summarized below:

1. The minimum radius for camber induced by cold bending in members up to a nominal depth of 30″ is between 10 and 14 times the depth of the member. Deeper members may require a larger minimum radius.
2. Cold bending may be used to provide sweep in members to practically any radius desired.
3. A length limit of 40 ft to 50 ft is practical.

Bending by heat is also a possibility, but this procedure is generally much more expensive than cold bending.

Note that providers for structural shape (including HSS) curving/bending often advertise their services in *Modern Steel Construction* and/or participate in the steel bending products listing (see the May 2003 issue). A curving or bending service supplier would be the best contact for determining minimum and maximum curved radii of specific shapes.

Keith Mueller, Ph.D.
AISC Steel Solutions Center
Chicago

PREVIOUS QUESTIONS

Do you have an answer? Send it to **solutions@aisc.org**.

PRYING ACTION

From January 2003 Steel Interchange

The 9th Edition *ASD Manual* illustrates procedures for bolted hanger-type connections with a single line of resistance to prying action on each side of the hanging member. If each line of resistance consists of a bolt group, what design and analysis methods should be used?

Jay Shniderman, P.E.
Van Nuys, CA

HEIGHT-THICKNESS RATIOS

From March 2002 Steel Interchange

Referring to *LRFD Specification* Sections F2.2, Appendix F2.2, and Appendix G.3:

For all of the standard rolled W-shapes, is the h/t_w ratio always less than 260? In other words, if a standard rolled shaped is being considered, is it necessary to check for the limit states of web shear yielding or bucking? Also, for all the standard rolled W- shapes utilizing up to 50 ksi specified minimum yield strength, is it always true that:

$$h/t_w \leq \frac{418}{\sqrt{F_y}}$$

Stephen Crockett
D. M. Berg Consultants, P.C.

HEIGHT LIMITATIONS IN OMFS

From March 2003 Steel Interchange

Why has the height limitation of 160 ft for OMFs in UBC 97 been reduced to 35 feet in the IBC 2000, for structural steelwork buildings in Seismic Design Category (SDC) *D*? I can't point to an exact reason, but commentary from some of the steel seismic seminars leads me to believe that AISC wants people use special frames of all types for almost everything (except maybe SDC *A* and *B*). I would expect the penalties to keep going for using ordinary frames in zone with moderate seismicity as well.